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Laterally Loaded Piles in Clay

By

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SUMMARY

The ultimate lateral resistance of a pile element moved horizontally can be analyzed by the theory of plasticity. At a certain depth the movements around the pile are purely horizontal and upper bound solutions can be estimated theoretically under undrained circumstances. Model tests in the laboratory show ultimate resistances close to the estimated limits and $p-y$ curves close to curves based on test results from full-scale piles. Rough and smooth piles with circular and square cross sections are investigated.

INTRODUCTION

Reasonable estimates of the behaviour of laterally loaded piles is an important factor in design and construction of off-shore installations.

To simplify the problem, the soil near the surface is considered to be pushed up in front of the pile, and in great depth the soil flow is assumed to be horizontal.

The relations between the horizontal soil resistance on a pile element p and the corresponding deflection y are normally described by $p-y$ curves which take the non-linearity of the soil pile interaction into account.

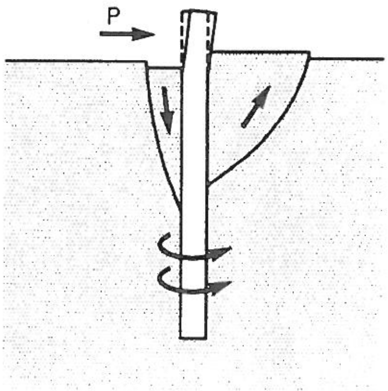

Fig. 1: Simplification of soil movement at lateral loading of pile, suggested by B. Broms in 1964 /1/.

P-Y CURVES

The $p-y$ relations are influenced by a great many factors including:

1. Installation
2. Geometry and pile properties
3. Rate and time of loading
4. Stress history
5. Stress-strain relations of soil
6. Depth below the surface

According to 2. the curves may be considered as analytical tolls and not as fundamental soil properties because the description also includes pile geometry and pile properties.

$p-y$ curves are normally related to the fundamental theory of soil properties, for example the ultimate lateral soil resistance p_u per unit length of pile. For the characteristic strain hardening "soft clay" ($c_u < 100$ kPa), /2 - 4/, p_u is found as the ultimate provided soil resistance.

ULTIMATE RESISTANCE

Introduction

In case of a deposit with undrained soil shear strength c_u constant with depth, the soil resistance near the surface will be less than the resistance in great depth, according to the difference in soil movements as mentioned above.

This paper deals with failure mechanisms in great depth where only horizontal movements take place.

Square piles

A square pile with width B is considered (Fig. 2). A possible mode of failure, which consists of both plastic zones and stiff bodies, is developed around the pile.

By an infinitesimal deflection of the pile in the loading direction, the rate of dissipation of energy within the deforming soil mass is

$$U_I = (3\,\pi + \sqrt{2} + 2) \cdot c_u \cdot B$$

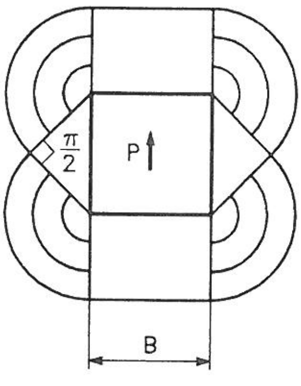

Fig. 2: Plane failure mode for a square pile.

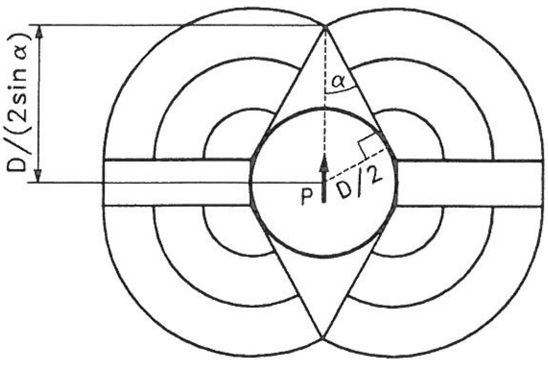

Fig. 3: Plane failure mode for a cylindrical pile.

If p_u is failure resistance per unit length of pile, the equation of work will give

$$p_u = 12.8 \cdot B \cdot c_u \tag{1}$$

and according to a pile with rough surface, p_u will be $16.3 \cdot B \cdot c_u$, assuming a perfectly rough surface of the pile.

Cylindrical piles

If the above presented failure mode is developed around a cylindrical pile with diameter D (Fig. 3) the dissipation energy may be expressed as

$$U_I = (2\,\pi + 4\,\alpha + 2 - 2\,cos\alpha + 2\,cot\alpha) \cdot D \cdot c_u \tag{2}$$

By minimizing equation (2) concerning α, the following result is obtained:

$$p_u = 11.9 \cdot c_u \cdot D \qquad (\alpha = 38.1°) \tag{3}$$

For a rough pile corresponding calculations will give $p_u = 12.9 \cdot c_u \cdot D$.

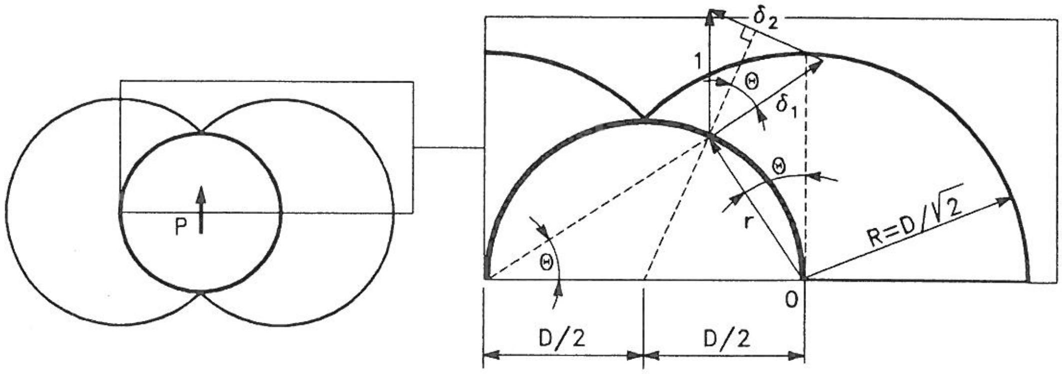

Fig. 4: Plane failure mode for a cylindrical pile.

Another failure mode developed around a smooth cylindrical pile is presented in Fig. 4. The movement of the pile is composed of a movement δ_1 perpendicular on the $r-$ direction and sliding δ_2 along the surface of the pile. Considering the angles the following result is obtained:

$$\begin{aligned} sin\,\Theta &= \frac{r}{D} \\ \delta_1 &= 2\,sin\,\Theta = \frac{2 \cdot r}{D} \\ \delta_2 &= 1 \end{aligned}$$

The soil moves as a stiff body because the movement δ_1 is proportional to r.

p_u can be expressed by the following equation:

$$p_u = \frac{D}{\sqrt{2}} \cdot \frac{3\,\pi}{4} \cdot c_u \cdot \sqrt{2} \cdot 4 = 9.42 \cdot c_u \cdot D \tag{4}$$

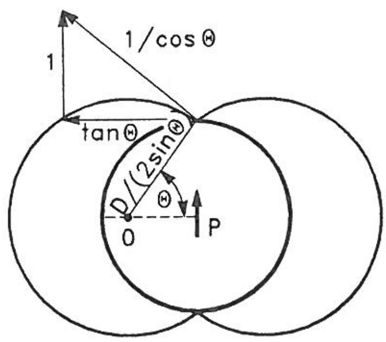

Fig. 5: Plane failure mode for a cylindrical pile.

and by integration around a rough pile, $p_u = 12.6 \cdot c_u \cdot D$.

To find a better solution the point of rotation O is moved (Fig. 5). For the failure mode composed of stiff bodies the ultimate resistance will be:

$$U_I = 2 \cdot c_u \cdot D \, \frac{(\pi - \Theta)}{cos\Theta sin\Theta} \tag{5}$$

and by minimizing equation (5) concerning Θ the following result is obtained:

$$p_u = 9.2 \cdot c_u \cdot D \qquad (\Theta = 51.3°) \tag{6}$$

It should be emphasized that the solutions here are based on a rigid, perfectly plastic response of the soil and it is assumed that the soil deforms at constant volume.

Limits

If the ultimate lateral soil resistance is nondimensionalized with respect to the soil strength and a characteristic dimension of the pile the following limits are found for the ultimate lateral soil resistance coefficient $N_p = p_u/(B \cdot c_u)$ or $N_p = p_u/(D \cdot c_u)$:

$N_p \, \epsilon$	[; 9.2]	smooth, circular piles
$N_p \, \epsilon$	[; 12.6]	rough, circular piles
$N_p \, \epsilon$	[10.3 ; 12.8]	smooth, square piles
$N_p \, \epsilon$	[10.3 ; 16.3]	rough, square piles

The lower approach for the square piles corresponds to a static solution with an infinite number of stress-strings in equilibrium.

LABORATORY MODEL TESTS

Performance of model tests

The present practice of constructing $p-y$ curves is based on test results from full-scale piles, which are very expensive with respect to research work. The idea was to find out whether it was possible to determine similar curves on the basis of model tests in the laboratory.

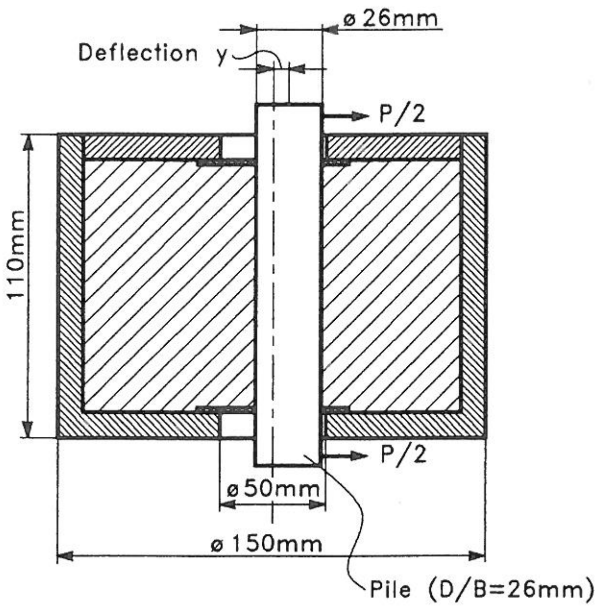

Fig. 6: Test specimen.

Trying to simulate the conditions in great depth, 42 tests were performed with rigid piles embedded in saturated clay till in small boxes (Fig. 6).

The undrained soil shear strength c_u was found in 3 - 4 places per sample by vane tests with a vane in miniature for the clay till used in the tests. All samples had a degree of saturation $S_w = 0.95 - 1$, a void ratio of $e = 0.43 - 0.45$, a density of $\gamma = 21.2 - 21.8\ kN/m^3$ and an undrained soil shear strength of $c_u = 50 - 90\ kPa$, according to the frequently defined "soft" clay ($c_u < 100\ kPa$).

Experiments were made with square and circular piles with smooth and rough surfaces.

All samples were consolidated at 240 kPa to obtain homogeneous conditions. After this process the lid was tightened and this stress state remained during the test by preventing the sample from dilating.

The piles were then loaded at both ends trying to obtain uniform soil resistance pr. unit length of pile. During the loading test measurements were collected every 2 sec. for 200 sec. The conditions were expected to be undrained.

Test results

One of the test results for a smooth circular pile is shown in Fig. 7 as a dimensionless $p - y$ curve.

Generally the curves obtained experimentally showed the same shape of the $p - y$ curves as the theoretically suggested ones but often there is inconsistency according to the residual resistance p_u. Normally people do not distinguish between different kinds of cross-sections, which is absolutely unreasonable.

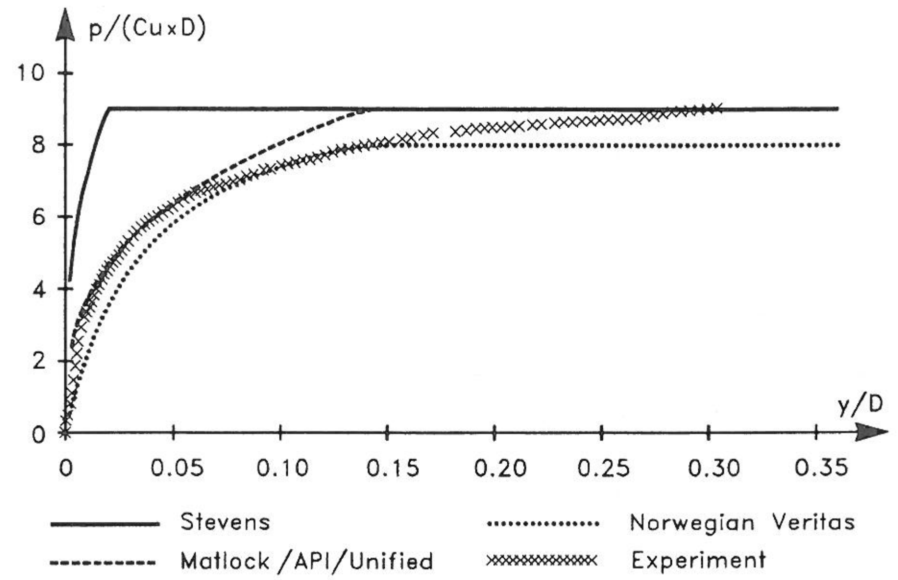

Fig. 7: Experimental $p-y$ curve for test no. 09033A. Smooth, circular pile. $c_u = 75\ kPa$, $S_w = 0.98$, $e = 0.44$, $w = 16\%$. *Theoretical curves: /2 - 5/.*

For the four different pile types the following mean factors were obtained

$N_p = 9.1$	smooth, circular piles	$N_p = 12.4$	smooth, square piles
$N_p = 11.5$	rough, circular piles	$N_p = 13.8$	rough, square piles

The experimental carrying capacities of soil for the respective pile types are situated between the previously determined theoretical limits.

All the experimental $p-y$ curves show a characteristic form corresponding to "soft clay", which means no weakening after rupture which is the case for stiffer, brittle clay. A single test was performed on a sample with $c_u = 160\ kPa$ and a similar curve was obtained. It could be expected that the form of the curve is not due to the undrained soil strength but due to the stress history, - normally consolidated or preconsolidated states.

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